CHAPTER 2
DESIGN OF ROCK REINFORCEMENT

2-1. General.

a. The design of reinforced rock structures follows the same basic steps used in the design of other structures. The differences, for example, between reinforced concrete design and reinforced rock design are in emphasis rather than basic design philosophy. The structural engineer in approaching the design of structures such as buildings and bridges uses conventional methods of structure analysis and provisions of design codes to produce a design which will perform as anticipated when it is loaded after completion. In these cases the modes of deformation and collapse of these structural configurations are well known. However, the methods of analysis and the provisions of design codes are not nearly so explicit when consideration is given to the behavior of the composite structure of steel, concrete, and rock, which is the actual case of a bridge or building and its foundations.

b. The discontinuous nature of rock masses permits many possible modes of deformation. Also, it should always be kept in mind that excavations in rock are made in a material that is always under in situ stress and strain and which generally is in stable equilibrium before the excavation is made. This is the opposite of most civil engineering structures where the structural materials are not fully loaded until the structure is completed and in service. The complexity of rock structures in a discontinuous rock mass has become apparent from experience with analysis techniques.

c. In the design of rock reinforcement, the primary emphasis should be to guard against the most probable modes of deformation that may lead to collapse. The information necessary to the design is not available in the early design stages but must be gathered from the time of preliminary geological investigations through the exploration, design, and construction stages of a project. The designer of rock reinforcement systems must place primary emphasis on modes of deformation rather than concentrating on calculations of stresses, strains, and load factors. Suitable construction procedures must also be considered as part of the design process and appropriate provisions made in the specifications to ensure that design requirements will be met. Also, the specification provisions must provide the contractual framework for modification to the basic design of the rock reinforcement as construction proceeds. It is important that the contractor is aware that such modifications will be made and this should be noted in the specifications.

d. The procedure to be followed in designing a rock reinforcement system should not be restricted to the reinforcement elements only but must also consider and be integrated with the overall design of the rock structure. In the following sections consideration is given first, to the several stages of design; second, to the basic characteristics of a sound design; third, to empirical guidelines that arise from past experience on other projects; and fourth, to analytical techniques that may be used to assist the designer.

2-2. Design Procedure.

a. General. It is beyond the scope of this manual to outline the considerations that lead to the need for an underground opening or an open cut of a given size and geometry in a given rock mass. The design procedure should, of course, be applied to each alternate configuration considered for a given project. It is assumed that the designer is confronted with the problem of designing a reinforced rock structure so that it maintains its stability under the service conditions to which it will be subjected.

b. Stages of Design.

(1) Preliminary design and estimate stage. The first design efforts should be directed to determining approximately the type and amount of reinforcement that might be required for a given project. At this point in the design the most useful information will be experience from similar jobs. Because the exploration and testing programs would not yet have provided the detailed information necessary for detailed analysis and design, the design engineer should become familiar with techniques of stabilization that have been successful. This familiarization should include a general knowledge of rock mechanics and rock stabilization which can be gained from text books, technical papers, and lectures. It should also include a review of the plans, specifications, and field experience for jobs with conditions similar to those expected on the project under consideration. Alternate types of reinforcement and schemes of excavation and reinforcement should be carefully outlined in preparation for final design.

(2) Final design stage.

(a) As geologic and rock engineering information becomes available and as the plan of the project is finalized, detailed design of the reinforcement system may be pursued. This detailed design has as its end product a set of plans and specifications which will indicate to the contractors what reinforcement the designer considers will be necessary to stabilize the rock structures. The design should include not only the number, length, size, and orientation of reinforcement elements but also excavation-reinforcement sequence and detailed installation requirements. Analyses of possible modes of deformation are made to the extent justified by the details known about the rock. Detailed study should be made of recent projects to ensure that better methods are not being overlooked. A series of laboratory and field tests should be performed to verify acceptability and practicality of all specified hardware and procedures. The specifications should also allow some flexibility in rock reinforcement requirements so that unanticipated geological conditions can be dealt with as economically as possible.

(b) A primary key to the success of a rock reinforcement system is the preparation of adequate provisions in the specifications. The specifications must serve not only to guide the contractor's work and quality control requirements but must also provide a means for informing both the contractor and the inspectors as to what the rock reinforcement requirements are for each project. Examples of quality control requirements included in specifications for a particular job are given by Smart and Friestad.[47] On some projects detailed study will have pinpointed zones requiring reinforcement in addition to the basic pattern. Such reinforcement should be designed and shown on the plans. No matter how detailed the geologic investigations may be, there will always be local conditions that cannot be foreseen and consequently will require additional reinforcement. The specifications should contain provisions for dealing with such conditions and paying for any additional reinforcement required. Guidance criteria to aid in deciding when to add the reinforcement should also be included.

(c) Instrumentation is a basic tool for monitoring rock behavior during construction and indicating variations from design assumptions. It should be planned and designed along with the basic excavation and reinforcement. Also, the specifications should indicate any interference with construction which the instrumentation program might cause. Contractor assistance with the instrumentation should be a pay item in the contract. The most meaningful and useful measurements in the past have been those recording rock deformation and movement. Extensometers, rock bolt deformeters, and survey reference points on the rock surfaces are the most common methods of monitoring rock mass deformations.

(3) Design modifications during construction. Requirements for rock reinforcement are not complete until the excavation is completed and all rock structures are stable. If maximum benefit is to accrue from flexibility in the specifications, then continuous checks on design assumptions should be made as construction proceeds. Signs of instability may call for further analysis and redesign based on modes of deformation not considered in the initial design and which can be

ascertained only through visual observation and measurements as the work proceeds. It is at this stage, that many analytical techniques may prove to be most useful. Modifications in the basic design that are made during construction may be of minor importance from the standpoint of construction cost but can be of major importance from the standpoint of overall stability.

c. Basic Characteristics of Sound Designs.

(1) Checklists. Checklists for the design of rock reinforcement systems are given below. The first includes desirable component activities of the design procedure, while the second gives desirable characteristics of the reinforcement system produced by the design.

(2) Design activities. The following activities should be an integral part of any design.

(a) Detailed geologic investigations. One of the designer's responsibilities is to request and obtain geologic data that are necessary for an adequate design. The collection of this data must begin very early in the design and gaps in the data must be allowed for by providing flexibility in the design procedure.

(b) Coordination between geologists and engineers. The design engineer must constantly review geologic data as they become available to check and modify assumptions made about the rock mass that were made when the preliminary designs were initiated (EM 1110-1-1801[1] and EM 1110-1-1806[2]). Also the geologists should be aware of design requirements so that they can supply geological data and interpretations which will be of maximum usefulness during both design and construction. In some cases the same geologists will be on the site during construction, which can be of considerable benefit to the designer and the resident engineer.

(c) Field and laboratory tests of reinforcement. If tests on the specified rock reinforcement installations are not performed prior to construction, problems may arise during the initial stages of excavation as the contractor applies the designer's specified installation procedures. This early period of construction is often very critical, for example, portal excavation at the beginning of a tunnel project. Construction problems with reinforcement installation at such critical times must be avoided. Consequently, the design engineers should develop a field testing program that includes drilling holes, element installation, and element testing. All details of such investigations should be recorded. Detailed procedures critical to the successful installation should be given special attention in the specifications. Such testing enables the designer to select the methods of rock reinforcement best suited to the site and to eliminate less favorable methods. It also provides bases for assessing unusual conditions that may arise during construction. Laboratory tests of hardware should be made a part of the design tests. Again it is emphasized that all details of tests should be recorded.

(d) Detailed study of case histories. The study of previous designs provides basic guidance on what has been found to be good and bad practice. However, studies of similar jobs cannot be limited only to bolting patterns, bolt lengths, and types. The geology and problems encountered during construction must be understood if the pitfalls at other projects are to be avoided.

(e) Analyses of probable mechanisms of deformation. The preconstruction design is not complete unless likely modes of rock deformation, including effects of hydrostatic pressure, have been investigated at least to the degree possible with available geologic data. Design details such as extra reinforcement at tunnel portals and intersections and in zones of highly fractured rock illustrate that such data have been considered so far as is practicable.

(3) Desirable characteristics of reinforcement systems. The plans and specifications should be checked to ascertain that the following desirable characteristics of reinforcement have been achieved to the maximum practical extent:

(a) Early installation of reinforcement. The behavior of rock masses under stresses induced by the excavation is one of strain-weakening in most cases where stability is in question. For this reason strains of an inelastic nature (permanent deformations) should be arrested as soon as practical following excavation in the case of an active construction project. In the case of natural slopes or existing structures, detection of such permanent deformations is basic to designing remedial measures to improve stability. The practicality of installing tensioned bolts immediately behind the working face in tunnels and recessed bolts and anchors through unexcavated rock has been proven. These practices should be followed in all cases where reinforcement is the primary means of rock stabilization.

(b) Ductility of the reinforcement elements. Ductility is critical to the successful use of rock reinforcement. Invariably there will be zones of rock that deform or yield with changing stress conditions. The reinforcement must be sufficiently ductile to accept reasonable deformations without failure. Common points of failure are in the anchorage and through the root of cut threads. Ideally, maximum use should be made of the ductility of the bar material itself. This means that the anchorage and bearing plate, washers, nut, and thread assemblies should have strengths greater than the yield point of the bolt shank. This may not always be achievable for the anchorage. However, full length bonding of the element to the rock by grouting will produce the ideal situation where failure at one point in the whole element assembly does not necessarily destroy its usefulness.

(c) Tensioning of bolts at the time of installation. The tension in a bolt at the time of installation combines with other factors including time of installation, time of grouting, and strength of the weakest part of a bolt to determine the effectiveness of the bolt assembly. If it is assumed that there are no parts weaker than the yield load of the bar or that the bar is bonded the full length of the drill hole, then the theoretically desirable tension in the bolt is the yield of the bar. This would achieve a maximum compressive stress being applied to the rock while still leaving all the post yield ductility of the steel available to accept rock deformation with constant or slightly increasing loads. However, if there are parts of this bolt assembly weaker than the yield strength of the bar and the bolt is not fully bonded to the rock, then the design is dependent on the reinforcement loads remaining in the elastic range. This is not sound rock reinforcement design. Also, the deformation necessary to increase the load in a 20-foot-long ungrouted rock bolt assembly from a working load of, say, three-quarters yield to full yield would be approximately 0.1 inch. If the deformation is concentrated between two rock blocks and the bolt is fully grouted much less deformation will bring the bolt assembly to yield. Under these conditions a rock reinforcement system should not be designed on the assumption that the elements will behave elastically. Experience has shown that specification of two-thirds to three-quarters of the yield load of the bolt assembly is a practical range for initial tension. This will provide a margin in the elastic range of the bolts to cope with variations in bolt installation and also provide a basis for realistically appraising the measurements from monitoring devices such as deformeters. These comments should not be construed to discount the use of untensioned anchors as rock reinforcement. Prereinforcement with recessed untensioned anchors may often prove to be the technically and economically desirable method of reinforcement. Untensioned anchors develop working loads as initial rock movements take place during excavation. The need for early installation and full length bonding of untensioned anchors cannot be overemphasized.

(d) Stable anchorage. The load and deformation conditions at the anchorage of a tensioned element are quite severe. This is particularly true of mechanical anchorages. High local stresses at the contact between anchorage and rock are conducive to both creep under sustained load and slip or partial failure under dynamic loading. Mechanical anchorages are more prone to relaxation than are grouted type anchorages. However, if the bond length is too short in grouted anchorages then slip may occur. After a bolt assembly is grouted full length, the likelihood of anchorage slip is very greatly reduced if not eliminated.

(e) Early full length bonding of elements to the rock. If the element is not fully grouted, there is always the possibility of loss of tension through anchorage or plate failure. Consequently, full length bonding of the element to the rock at the earliest practicable time provides assurance of the effectiveness of reinforcement during the critical period of nearby excavation. Damage to grout surrounding the reinforcement element from blasting is usually minimal, and the effectiveness of fully grouted reinforcement during this critical period in preventing rock movement is of primary consideration.

(f) Surface treatment. It is seldom possible to install reinforcement through each rock block exposed by the excavation, particularly if the rock is closely jointed. For this reason supplemental surface treatment is required to restrain the rock surface and prevent raveling that could lead to local fallout and possibly general fallout. This is particularly true in crown areas of underground excavations. Surface treatment includes the provision of chain link or welded wire fabric, strapping, and shotcrete. This treatment not only contributes to the structural effectiveness of a reinforcement system but also provides safer working conditions, particularly from rock fall. Early installation of surface support such as chainlink fabric may result in damage to the fabric by flyrock. However, this damage is more than offset by the advantages from its use.

(g) Quality control provisions in the specifications. Even though the designer may have provided appropriate methods of reinforcement and validated his specified procedures by field testing, the contractor's performance of specified installation must be checked. The specifications should require a test program prior to production installation of reinforcement that will verify that proper techniques are being used by the contractor's work force to install the reinforcement. Quantitative indicators of satisfactory installation should be included in the specification for the benefit of inspectors and the contractor. Pull testing of bolts and full flow return of grout as an indication of complete grouting of a bolt are examples of such indicators.

d. Empirical Guidelines for Sound Designs. A summary of many important rock reinforcement case histories is included in chapter 7. The final design of these projects provides the basis for the development of empirical rules that may be used as a guide for minimum reinforcement to be included in preliminary designs. Detailed analyses taking account of geological information derived from diamond drill cores, exploratory tunnels, and surface mapping, as well as results from laboratory and in situ testing to determine rock behavior characteristics, usually indicate the need for more reinforcement than called for by such rules. On the basis of data given in chapter 7, several empirical rules are presented in tables 2-1 and 2-2. It must be reemphasized that these rules give a preliminary configuration for rock reinforcements which must be checked, analyzed, and, as necessary, modified to meet the requirements of a specific rock reinforcement design.

e. Analytical Techniques for Rock Reinforcement Design.

(1) The analytical methods used for assessing the stability of rock structures are direct developments from structural analysis and applied mechanics. Their complexity ranges from the simple case of a block sliding on a surface of known frictional resistance to highly complex finite element solutions that include the effects of slippage along discontinuities and fracture of rock blocks. The analysis of the stability of a rock structure requires the behavior of the structure to be stated in terms of its geometry, the load deformation characteristics of its materials, the virgin in situ stresses, the geological characteristics of the rock, and the conditions induced by the excavation. Such statements may range from a simple case, such as a rock block on the surface under gravity load, to the highly indeterminate conditions associated with intersections of underground openings. The usefulness of an analysis is determined not by the arithmetical accuracy of the calculations but by the accuracy of the input data mentioned above. Various mathematical models that have been used to analyze reinforced rock structures and their applications are discussed below.

(2) Elastic analyses.

(a) Stress concentrations around openings. Solutions for calculating stress conditions near single and multiple openings in stressed elastic media are available for several simple shapes. These include circular or elliptical shapes; and square, rectangular, and triangular shapes with rounded corners as presented by Jaeger and Cook[34] and Obert and Duvall.[38] If the virgin in situ state of stress prior to excavation is known, then the theoretical stresses near the cavern walls can be calculated. These are generally the most important as failure begins at the new surface of an excavation. Comparisons of stresses and the rock strength parameters give a quick indication of areas where stability problems may exist.

(b) Finite element solutions. Elastic finite element analyses have been used to study the stress patterns around single and multiple openings of complicated geometry and in media of varying elastic

Table 2-1. Minimum Length and Maximum Spacing for Rock Reinforcement

Parameter	Empirical Rules	Notes
Minimum Length	Greatest of:	
	a. Two times the bolt spacing	
	b. Three times the width of critical and potentially unstable rock blocks*	
	c. For elements above the springline:	
	1. Spans less than 20 ft - 1/2 span	
	2. Spans from 60 ft to 100 ft - 1/4 span	
	3. Spans 20 ft to 60 ft - interpolate between 10-ft and 15-ft lengths, respectively.	
	d. For elements below the springline:	
	1. For openings less than 60 ft high - use lengths as determined in c. above	
	2. For openings greater than 60 ft high - 1/5 the height	
Maximum Spacing	Least of:	
	a. 1/2 the bolt length	
	b. 1-1/2 the width of critical and potentially unstable rock blocks*	
	c. 6 ft	Greater spacing than 6 ft would make attachment of surface treatment such as chain link fabric difficult
Minimum Spacing	3 to 4 ft	

* Where the joint spacing is close and the span is relatively large, the superposition of two bolting patterns may be appropriate; e.g., long heavy bolts on wide centers to support the span and shorter and thinner bolts on closer centers to stabilize the surface against ravelling due to close jointing as outlined by Reed.[44]

Table 2-2. Minimum Average Confining Pressure for Rock Reinforcement

Parameter	Empirical Rules	Notes
Minimum Average Confining Pressure at Yield Point of Elements	Greatest of:	This assumes the elements will behave in a ductile manner.
	I. Above Springline --	
	a. Pressure equal to a vertical rock load of 0.20 times the opening width	a. For example if the unit weight of the rock is 144 pcf and the opening span is 75 ft the internal confining pressure is 15 psi.
	b. 6 psi	b. For the maximum spacing of 6 ft this requires a yield strength of approximately 32,000 lb.
	II. Below Springline --	
	a. Pressure equal to a vertical rock load of 0.1 times the opening height	a. For example if the unit weight of the rock is 160 pcf and the cavity height is 144 ft the required confining pressure is 16 psi.
	b. 6 psi	b. See note b. under I above.
	III. At Intersections	
	a. 2 times the confining pressure as determined above	a. This reinforcement should be installed from the first opening excavated prior to forming the intersection. Stress concentrations are generally higher at intersections, and rock blocks are free to move toward both openings.

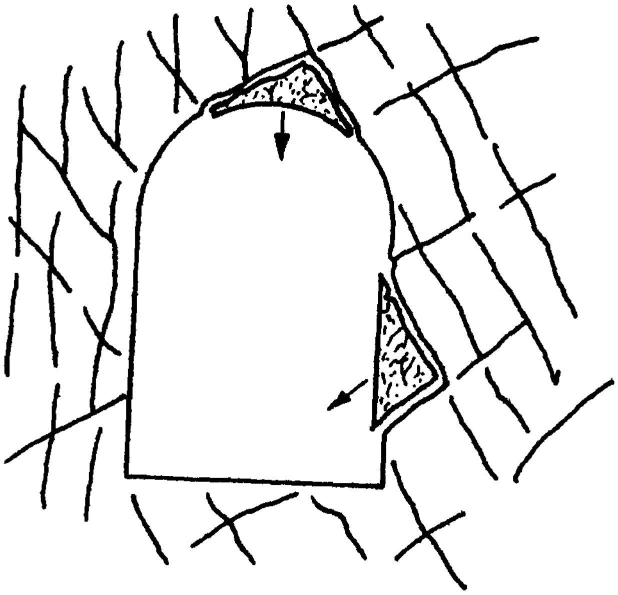
properties. From such analyses areas of high compressive stresses as well as tensile stresses can be delineated and the reinforcement planned accordingly. The Churchill Falls project is a recent example of such analysis.[27] Where the stresses resulting from rock bolting have been included in the analysis, they appear to have only a small effect on the overall stress pattern around openings. In many cases finite element analyses without including the bolting forces are sufficient to outline potential problem areas.

(c) Photoelastic methods. Photoelastic studies have provided much the same information as elastic finite element studies. Photoelastic methods predate finite element work and have been used primarily for homogeneous isotropic materials. However, limited studies of discontinuous and layered media have been made. It may be pointed out that the results of stress analyses either by photoelasticity or numerical methods combined with studies of case histories provide a good source of qualitative and often quantitative information for new designs.

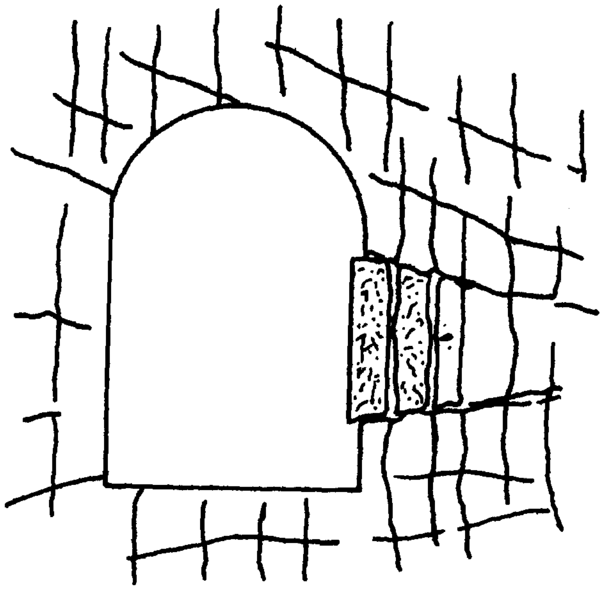
(3) Limit analyses. The most valuable analyses used in the design of rock reinforcement are those that consider possible modes of deformation and methods for arresting such deformation prior to collapse of a given rock structure. Such analyses are studies of failure mechanisms. In this respect they are similar to limit (or plastic) design of steel and reinforced concrete structures. This approach assumes that "yield" can occur at certain points without total collapse of the structure. This approach to rock structure stability realistically accounts for the behavior of the rock and the reinforcement. In rock structures, just as in steel or concrete structures, it is not always possible to keep all stresses at all times less than the intrinsic "strengths" of the materials. However, it is a matter of experience that many excavations where the rock around the opening is highly fractured (that is, it has "failed") are stable and have not collapsed. In such cases it is essential to know that overall deformations of the excavation are "stable." This often requires measures to be taken to improve the rock mass behavior by means such as grouting and rock reinforcement. The following methods are useful tools available for analysis of rock structures:

(a) Rock block stability. In any excavation, the force of gravity cannot be ignored when considering the forces which act on excavation surfaces. Specifically, gravity is a direct contributor to stability or instability immediately around the surface of an excavation, where relocation and permanent deformation has already taken place.

1. As illustrated in figure 2-1, slippage along joints could cause individual rock blocks to become separated from the main rock mass.[35]



(a) Fallout of blocks isolated from rock mass due to failure along joints.



(b) Progressive partial failure of joint blocks adjacent to excavation surface.

Figure 2-1. Gravity effects on jointed rock stability.

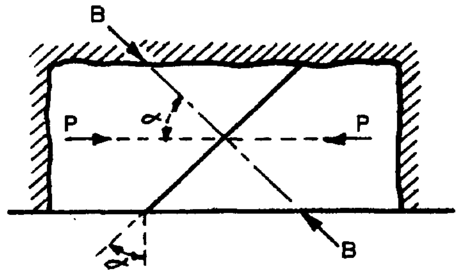
Factors which would induce such conditions are (1) the irregularities of joints are nominal, (2) the resistance force against sliding along joints is low, (3) the angle the joints make with the surface of the excavation is small, and (4) the force of gravity tends to induce motion of the block. Blocks in the roof may be entirely free to fall, but blocks in the wall would have to slide or rotate along the joints at its sides and base before fallout could occur. It is obvious that similar conditions in the floor would not cause concern, but in zones of high stresses or swelling ground the floor can heave into the opening.

2. Calculations to determine the effect of rock reinforcement on the movement of simple systems of rigid rock blocks are not difficult. Several simplified cases are shown in figure 2-2.[37] The expressions shown for each case indicate the required bolting force to maintain stability assuming cohesion along the joint is zero and that slippage along the joint is physically possible. The force of gravity in figure 2-2(a) and (b) is ignored. It is important to note that the forces P are not necessarily the result of elastic behavior in the rock mass, since small deformations across the discontinuities may reduce P to considerably less than what would be assumed from elastic analysis. The analyses of discrete blocks that may be formed by persistent discontinuities should always be analyzed even if definitive tests of in situ rock properties and positive verification of the existence of these discontinuities has not been made. Additional reinforcement to stabilize such blocks is usually required beyond that needed for general overall pattern reinforcement.

3. Similar analytical models to those in figure 2-2 may be postulated to take into account failure by rotation of rock blocks. Rotation as indicated in figure 2-3 almost always plays an important part in deformation and failure mechanisms in rock structures.

4. Sliding rock block models are the most practical method of analysis of rock slopes. Methods for analyzing such models are presented by Hendron, Cording, and Aiyer.[14] The analysis of rock slopes includes consideration of all probable sliding blocks and the possible directions of sliding. As is illustrated in the above reference, though the geometry and statics may be quite complicated in such analyses, the basic approach is simply one of rigid blocks sliding on failure planes of known resistance.

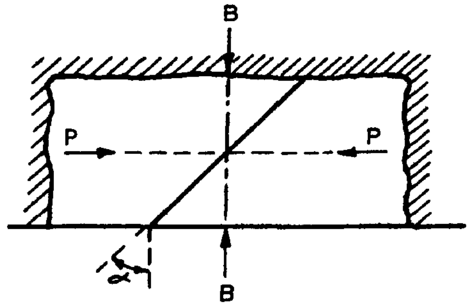
(b) Rock beam or slab concept. In order to gain a better understanding of rock behavior the simple case of flexure in a beam or slab can be considered. Where excavations are made in stratified rock, this concept is directly applicable. In a fixed end beam or slab or substantially uniform material, such as some rock or concrete where the



For stability:

$$\frac{B}{P} > \sin \alpha(\cot \phi - \cot \alpha)$$

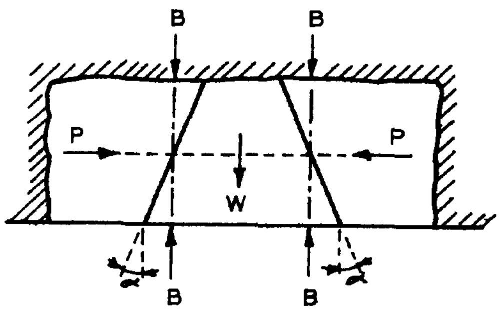
(a) Single joint with bolt normal to joint.



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P} < \tan(\alpha + \phi)$$

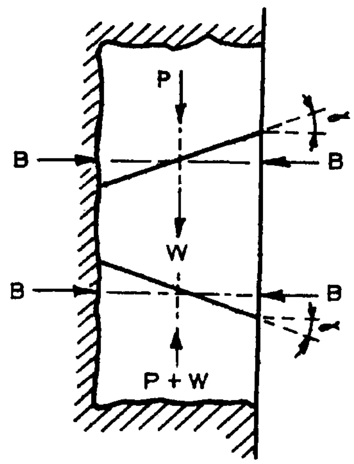
(b) Single joint with bolt normal to end load.



For stability:

$$\tan(\alpha - \phi) < \frac{B - 1/2W}{P} < \tan(\alpha + \phi)$$

(c) Block in horizontal surface.



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P + 1/2W} < \tan(\alpha + \phi)$$

(d) Block in vertical surface.

B = Force exerted by bolt
P = Direct force on joint
$\tan \phi$ = Coefficient of joint friction
W = Weight of block

Figure 2-2. Simple rock bolt models.

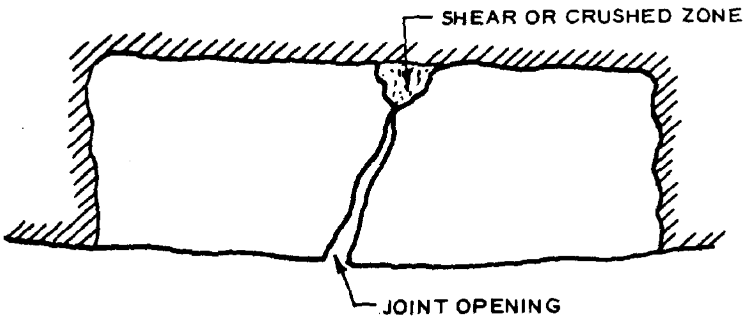


Figure 2-3. Failure by rotation.

tensile strength is less than the compressive strength, the mode of deformation and failure under increasing load will be as sketched in figure 2-4. Cracks will appear first at the ends, A and B, where the flexural tensile stresses are highest and then at the bottom in the center, C. This type of behavior is particularly apparent in materials which not only are stratified but also have joints, shears, or planes of weakness transverse to the axis of the beam. With increasing deformation there will be a tendency for one or more cracks near the center of the span to become "preferred." This leads to a condition as shown in the idealized sketch in figure 2-5. The beam, with increasing

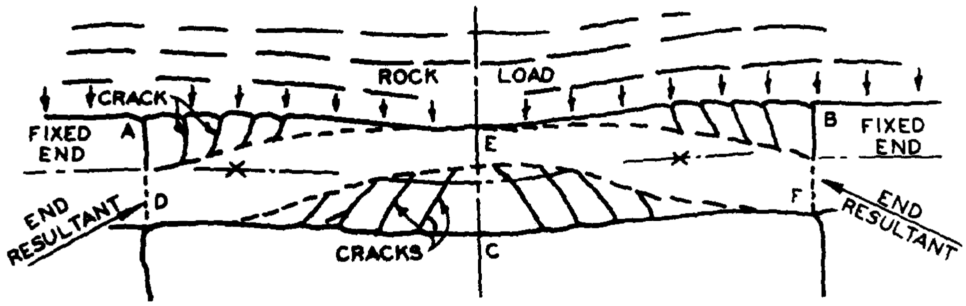


Figure 2-4. Mode of failure of uniform material beam.

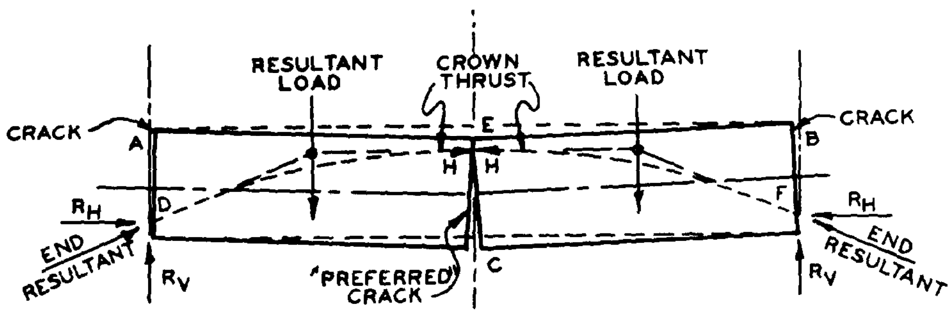


Figure 2-5. Idealized sketch of beam behavior.

deflection, is rotating about a bearing area at the ends, D and F, with a "hinge" at E, thus virtually forming a three hinged arch. With relatively rigid abutments this action leads to large horizontal reactions R_H at D and F, and a horizontal thrust at E. Failure will occur by crushing and shearing of the rock at D, F, and E, which will give increased deflections and ultimately lead to collapse. In practice the abutments at D and F are not rigid and do deform. This has the effect of increasing the central deflection and also of giving a larger bearing area at D than at the center top at E. Wright and Mirza[55] have investigated the stress distribution about such cracked beams photoelastically, and have determined that the bearing area at E is only about 18 percent of the depth of the beam. It is obvious that if the depth of such a beam is small relative to the span then the "arch" action described cannot be effective and collapse will take place with very small deflections. The design of reinforcement to inhibit this type of failure is discussed in the next section.

(c) Rock beam reinforcement. The earlier applications of rock reinforcement were mainly in mining work in sedimentary strata and gave rise to the concept that rock bolts created a beam or slab by clamping together a number of thin or incompetent horizontal strata. Rock reinforcement creates a structural member in any jointed rock mass if a systematic pattern of bolts is used (figure 2-6). The bolts, if tensioned, create a zone of uniform compression somewhat shorter in thickness than the length of the bolts. This zone is confined and acts effectively in stabilizing the rock excavations. Where untensioned grouted rebar is used instead of tensioned rock bolts a somewhat similar condition also develops after limited deformation has taken place. Such reinforcement of a beam or slab roof is sketched in figure 2-7a. The use of steel strapping rockbolt ties or steel channels under the bearing plates of the bolts (figure 2-7b) leads to the concept of a composite beam or slab with the steel channel acting as the tension

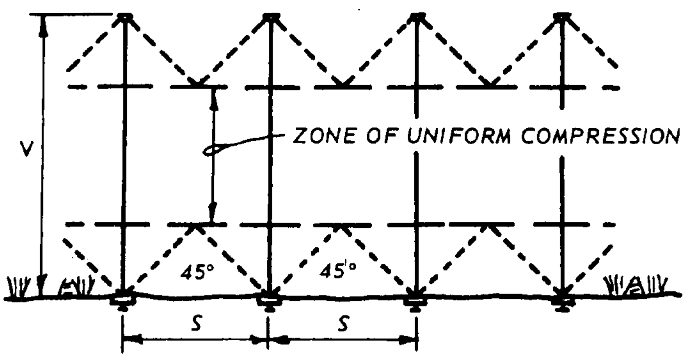


Figure 2-6. Structural member concept.

component. Undoubtedly the beam or slab tends to act at least partially as fixed ended, and angling the bolts near the supports as shown in figure 2-7_c_ will increase their effectiveness. The construction stability that may be made by surface treatment such as steel straps, wire mesh, or a combination of shotcrete and wire mesh, if it is considered as tensile reinforcement at the bottom of the beam, can be assessed by using the archway reinforced concrete beam theory, Lang.[36]

1. In addition to "knitting" together the jointed rock layer between the ends of the bolts and increasing the basic shear strength of the rock in this layer, the rock bolts also act as shear or diagonal tension reinforcement for this layer considered as a beam or slab. Where steel channels or ties are used with angle bolts (figure 2-7_c_) the action is analogous to post-tensioning in reinforced concrete practice.

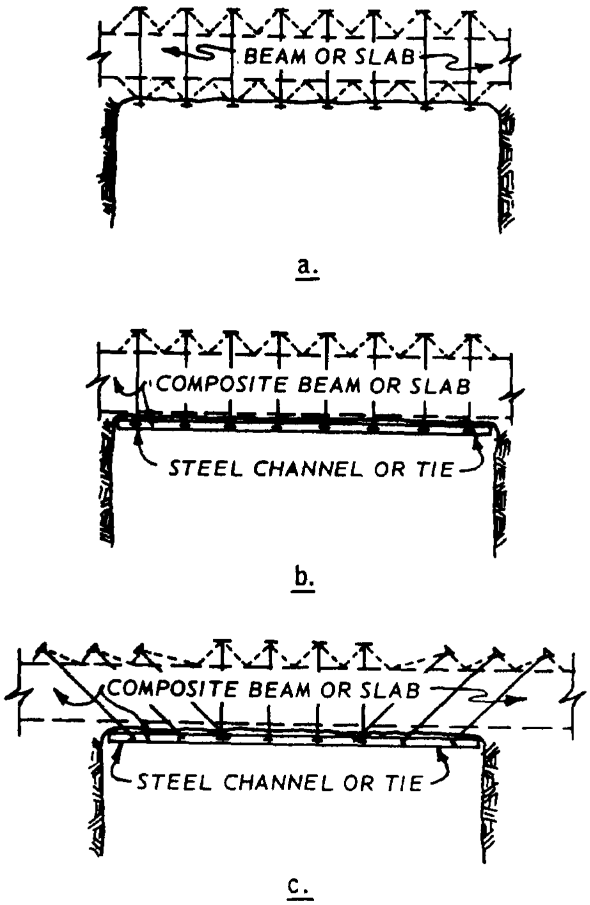


Figure 2-7. Beam or slab concept.

2. The length of the rock bolts is related not only to the geological features of the rock near the surface but also to the span of the opening. The structural member created by the bolts near the surface should be relatively deep compared to the span. It is also related to the spacing chosen for the bolt pattern. Due consideration must be given the type and condition of the rock that is being reinforced.

3. The analysis concept indicated above need not be limited to the crown of a rectangular underground chamber. Similar beam or slab action could be developed horizontally on the face of an open excavation or the walls of an underground excavation. Local instability resulting from rock block rotation as shown in figure 2-3 lends itself to similar analysis. In checking any existing or contemplated rock reinforcement pattern it should be kept in mind that beam action is not likely to occur alone. The rock beam may also be loaded axially as a column.

Axial loading will influence the formation of diagonal tension cracks and introduce the possibility of buckling.

(d) Arch reinforcement. In tunnels or curved roof excavations, rock reinforcement stabilizes the roof by creating a structural arch within the rock between the ends of the bolts. Typical examples are shown in figure 2-8. The effect on the arch member of varying the length and spacing of the bolts is also illustrated. In cases where

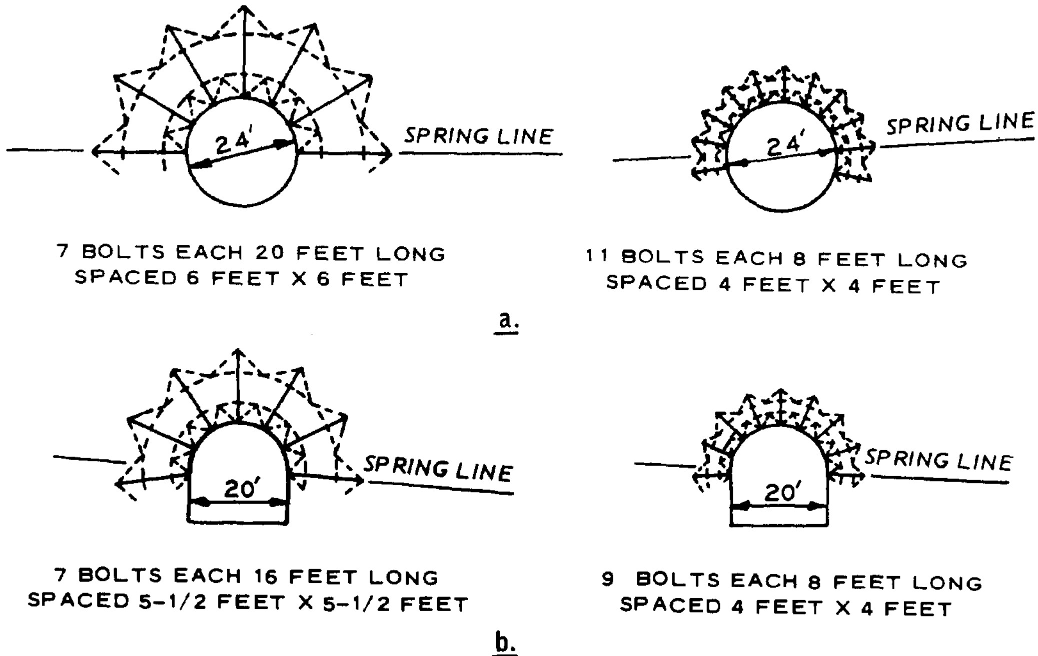


Figure 2.8. Arch concept of rock reinforcement.

the occurrence of persistent well defined joints requires the use of relatively long bolts, it may be feasible to use a smaller number of these and provide shorter supplementary rock bolts between the longer bolts, as shown in figure 2-9. This creates a more heavily reinforced zone near the surface and is effective in stabilizing closely fractured rock.[36] Such shorter bolts can also be used to "split" a regular pattern of primary bolts where monitoring has shown extra reinforcement to be necessary.

1. As in the case of the reinforced rock beam the thickness of the arch should be much larger relative to the span than is considered normal in reinforced concrete or masonry arches. In most cases a static analysis of the "effective arch" inside the reinforced area of the rock will show whether relatively high stresses or possible flexure of the

ROCK REINFORCEMENT

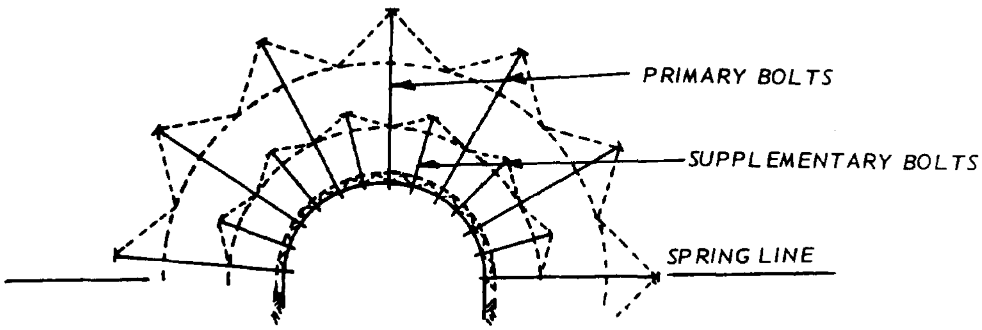


Figure 2-9. Supplementary rock reinforcement.

arch are possible. If the rock is stratified or has a system of joints cutting the effective arch, then shear on such planes of weakness should be checked.

2. With an arch type of roof in an excavation special attention should be given to the abutment or haunch areas. In many cases it will be found that longer bolts are required in this area than may be needed in the crown. The results of an elastic analysis would indicate the likelihood of high stress concentrations or tension zones in these areas as well as the walls and other parts of the excavation. Special analysis of reinforcement requirements in such areas may be required.

(e) Elastoplastic deformation analyses. Rock in situ before it is disturbed by the excavation of a tunnel or other opening is in equilibrium with the virgin in situ stresses. Following excavations stress concentrations are induced around the opening and at the new surface the principal stresses perpendicular to the surface are zero and the stresses tangential to the surface are the maximum principal stresses, with magnitudes which depend on the geometry of the opening and of the virgin in situ stress field. If the stresses tangential to the surface exceed the unconfined compressive strength of the rock, then even in an intact rock, failure will occur. If the rock is jointed or has other planes of weakness intersecting the new surface, then failure will occur and migrate from the surface into the rock mass. Initially, elastic deformation will occur, followed by permanent or plastic deformation. Using the Coulomb criterion for failure, it is possible, if the cohesion, angle of internal friction, and other rock parameters are known, to calculate the thickness of the plastic deformation zone where Coulomb criteria hold as well as the location of the boundary between this zone and the elastic deformation zone where elastic conditions prevail.

1. The theory has been given by Jaeger and Cook[34] and available closed-form solutions reviewed by Hendron and Aiyer.[11] Design approaches have also been investigated by Goodman and Dubois.[10] Existing solutions are for circular tunnels only, under conditions of hydrostatic virgin in situ stresses. However, such analyses for circular tunnel behavior are quite valuable in the design of any tunnel or other excavation with an arch-shaped roof and have been used as a basis for rock reinforcement design by Talobre[48] and others. These analyses allow calculation of the pressure needed on the surface of the excavation to stabilize the relaxed or plastic deformation zone around the excavation and prevent continuing migration of this zone away from the excavation. Theoretically, such a stabilizing pressure can be supplied by a system of rock reinforcement and the bolts anchored beyond the plastic deformation zone.

2. Shorter bolts that are not anchored beyond the plastic deformation can also improve conditions in the rock near the opening thereby achieving additional stability of the plastic deformation zone. The reinforced material is in triaxial compression rather than unconfined compression which occurs at the surface and hence is basically stronger. Also the reinforcement system, although not applying sufficient pressure to prevent the plastic deformation zone from forming, does prevent ravelling and fallout from the surface and thus inhibits "stoping" action that otherwise would take place. Consequently, surface treatment becomes of extreme importance. It may be noted that gravity effects on relaxed rock in the roof of excavations have also been approximated in these analyses.

(f) Finite element analysis. The finite element methods of analysis as well as the elastic analyses mentioned earlier, can be applied to simulate jointed rock consisting essentially of discrete blocks. In the more sophisticated models, predominant joint sets or other possible planes of weakness can be approximated and account taken of failure along discontinuities (joints, etc.) as well as the elastic behavior of the individual rock blocks and different physical properties for various elements. However, the detailed delineation of rock properties throughout a large area and the very large computer capacity required to cope with all these items in an underground complex limits the usefulness of such analyses as design tools. Emphasis in the past has been on models of an entire rock mass in a slope or around an opening using two-dimensional models. Three-dimensional models have been limited to simple axisymmetric cases. However, there appears to be some promise of using three-dimensional finite element analysis to examine local conditions in projects under construction where initial behavior of the rock is known and can be used as a check on the adequacy of the program to predict further behavior. At present, these methods of analysis are useful supplementary design tools where the magnitude of the job warrants the expense and time involved. Examples of recent work in finite element analysis are presented by Goodman[9] and Heuze and Goodman.[13,32]

(g) Interactive computer graphics. The use of interactive graphics for the input and output of geometrical data can be applied to support systems for rock slopes and tunnels. A computer program can model the behavior of assemblages of rock blocks and visually display this behavior on the screen of a Cathode Ray Tube (CRT). There are no restrictions on block shapes and no limits to the magnitude of displacement and rotations that are allowed. The user specifies the rock geometry by drawing lines on the CRT. This information is passed to a minicomputer which interprets each closed area as a discrete block and allows the blocks to move relative to one another under the action of gravity and user specified forces. Joint surface properties (maximum of ten values) may be specified and individual blocks may be excavated, fixed in place, or released as the program runs. The main assumption built into the program is that all deformations occur at the block surfaces. Contact forces can be displayed both in numbers and vectors. As the user may create vector forces to act upon or stabilize any particular block, the size, length, and direction of load (e.g. rock bolts) necessary to stabilize a rock slope or tunnel roof may be determined. The user may experiment with various patterns of rock bolts to determine the most effective distribution to stabilize a particular rock structure. The Distinct Element Method utilizing interactive graphics is useful for modeling numerically those rock systems for which the underlying mechanisms are not known. This system may be treated as a physical model having the additional advantage of being able to vary any parameter on demand. Methods and examples of computing tunnel supports are given by Cundall.[8]

(4) Physical modeling and pilot projects. As an aid to the designer, physical models of the rock structure to be reinforced may be tested under laboratory conditions. Quite simple models can often give a key to potential modes of behavior and failure. Qualitative simulation of rock reinforcement can also be introduced and provide a guide to the need for reinforcement in critical areas. Quantitatively, their usefulness is limited not only by the geologic information available but also by the difficulties of proper scale modeling of the rock properties. On large projects where exploratory tunnels are constructed in the project area, scale models of the large excavations can sometimes be made and serve to test both excavation and reinforcement procedures. Such tests, which give confidence to the designers and the contractors that the design is both technically sound and practical, have been reported by Endersbee and Hofto[30] for the Poatina hydroelectric project in

Tasmania, Australia. Physical model tests are also useful for checking the validity of mathematical model results or for providing a better understanding of the deformation behavior of the reinforcement, the rock, and the discontinuities in the rock. Such tests are reported on by Bureau, Goodman and Heuze.[12]